MIL07

Via Lombardia 1, Siziano, MI, Italy

STACK INFRASTRUCTURE

Calculation - Rainwater and Wastewater - Campus

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1. SCOPE OF WORK

The primary objective of this report is to outline the design philosophy and sizing criteria for the proposed stormwater and wastewater infrastructure networks. For areas subject to the requirements stipulated in Regional Regulation No. 4 of March 24, 2006 (pertaining to first-flush treatment), the planned approach involves the separation and treatment of rainwater in accordance with the regulation's provisions. For a more detailed discussion on the design and verification of stormwater management works, please refer to the dedicated document MIL07-DBA-EX-XX-RP-C-52001, which has been prepared in compliance with the requirements set forth in R.R. No. 7 of November 23, 2017 and subsequent amendments.

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2. DESCRIPTION OF THE DESIGNED HYDRAULIC NETWORKS

The project comprises the following hydraulic drainage networks:

- stormwater runoff management network for roads and courtyard;
- stormwater runoff management network for roof drainage;
- wastewater management network.

The stormwater management network for roads and courtyard collects water through interception systems such as storm drains and grated gutters. Along the internal roadway, the collection system is provided by a single-slope road cross-section, with a gully inlet installed at the low point, as shown in Figure 2.1. The only exception is at the parking areas, which are graded with a reverse slope towards a catch basin located on the outer side of the road.

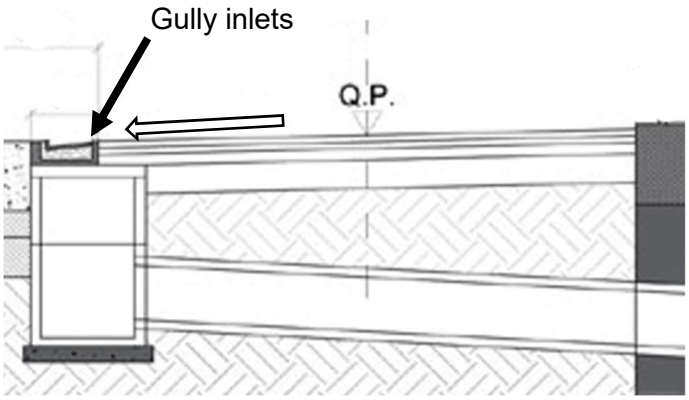

Figure 2.1: Schematic representation of road section

The storm drains connect to a pipeline, positioned centrally within the roadway whenever the interference with other underground utilities allows it.

The average longitudinal slope of the drainage network has been assumed as constant across the entire site, equal to 0.3%. The ground surface is currently assumed to be flat, and the design has been developed using relative elevations referenced to the existing ground (pending confirmation of absolute site benchmarks and roadway longitudinal profiles). It is recommended to provide a minimum roadway longitudinal grade greater than 0.3%, crowned at the northern origin of the trunk lines and falling toward the detention tanks at the southern boundary. Where a continuous longitudinal grade cannot be achieved, local longitudinal slopes should be formed toward each inlet and grated gutter to ensure effective drainage.

The drainage system for roads and courtyard consists of three main trunks developing around the data center – northwest, west, and south. The northwest and south trunks discharge to a terminal manhole in the southeast sector, while the west trunk discharges to a terminal manhole in the southwest sector.

Although not subject to Regional Regulation 4/2006 (first-flush requirements), each terminal manhole feeds a pretreatment system consisting of a continuous in-line desander and oil separator. After pretreatment, detention will be ensured by two underground concrete tanks: DT1 (southwest), receiving runoff from the northeast and south trunks, and DT2 (southeast), receiving runoff from the west trunk.

Refer to the dedicated stormwater management report MIL07-DBA-EX-XX-RP-C-52001 for the sizing and verification of this works.

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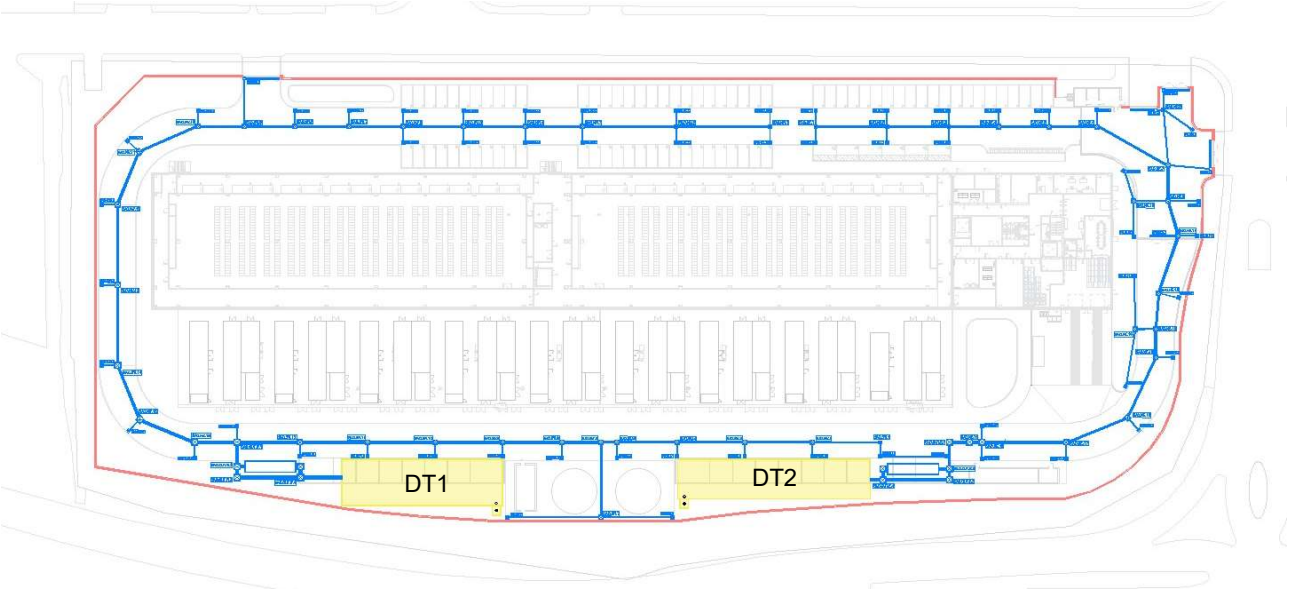

Figure 2.2: Plan view of the streets and yards stormwater network — connection to DT1 and DT2 detention tanks

The stormwater runoff from the generators and transformers areas are subject to the requirements of Regional Regulation 4/2006 of the Lombardy Region, as the water is considered potentially oily and requires first-flush treatment. Based on the information provided by the Client, no waste storage areas or other areas potentially subject to first-flush requirements have been identified. Although the loading/unloading bay is not subject to the requirements of Regional Regulation 4/2006 of the Lombardy Region, it has been included in the potentially oil-contaminated drainage network due to site topography.

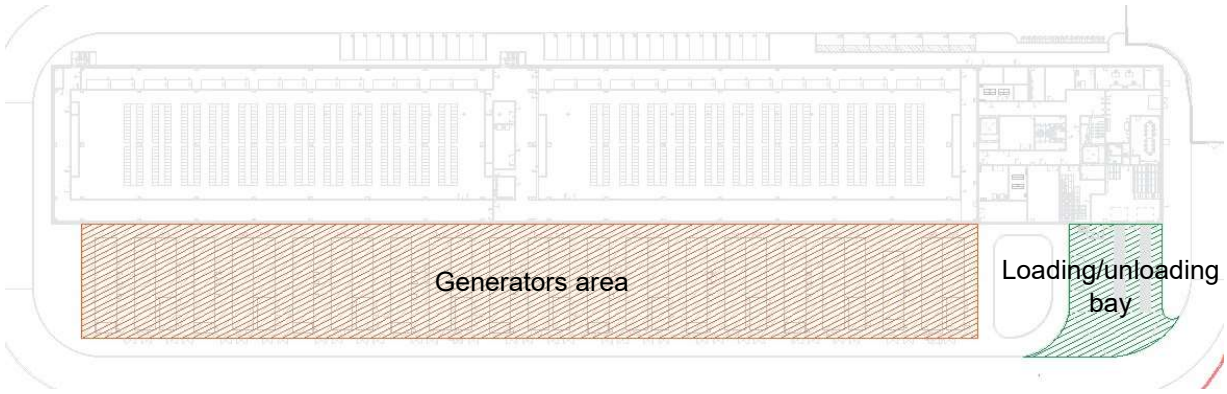

Figure 2.3: Areas subject to Regional Regulation 4/2006 (generators) and loading/unloading bay.

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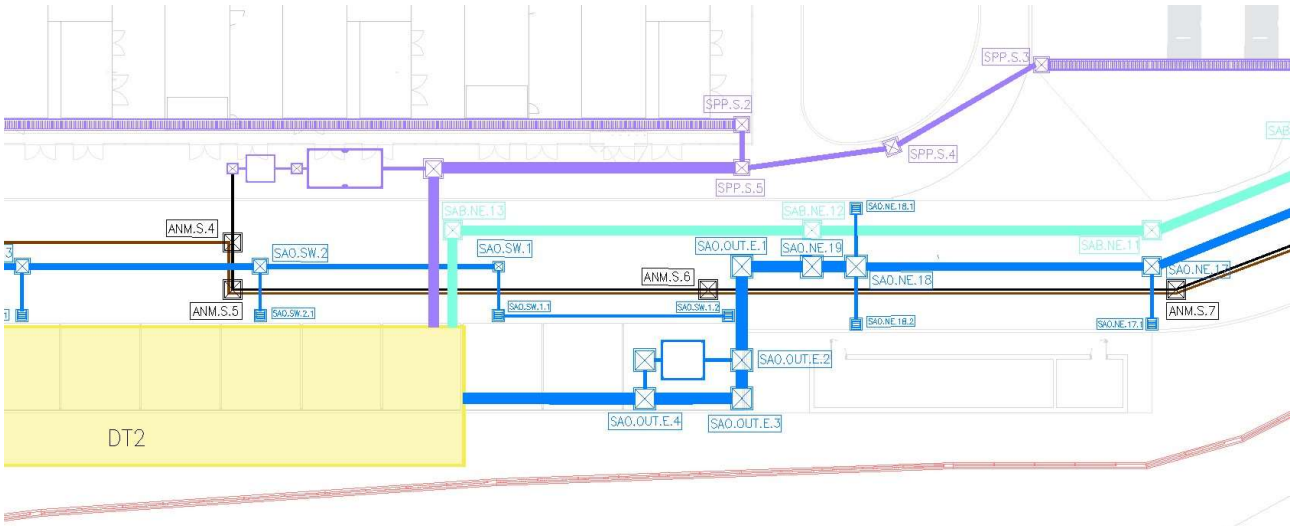

Figure 2.4: Design plan sketch of the first-flush treatment system for areas under Regional Regulation 4/2006 and loading/unloading bay

The generator area comprises a single-sloped reinforced-concrete base slab, directing runoff southward to a single grated gutter integrated along the southern edge. The in-situ gutter is cast integrally with the slab.

Runoff from the generator-yard gutter and from the loading-bay grated inlet is collected into a single dedicated network for potentially oily stormwater. Because the loading bay lies at an elevation roughly 2 m below the surrounding yards, the drainage layout has been designed around this constraint.

The potentially oily stormwater network discharges to a first-flush diversion manhole located upstream of treatment, where the first flush is separated, pretreated (desander and oil separator), and then discharged to the public sewer on Via Liguria, to the north-east of the site. The residual flow is conveyed directly to the DT2 detention tank.

The stormwater collection network from the building roofs originates from the downspout columns located along the north and south façades of the main project building. Pending receipt of the roof layout and exact downspout locations, these have been assumed for this stage. No glycol risk has been reported by the Client, and the network is not subject to Regional Regulation No. 4/2006; accordingly, no first-flush isolation or pretreatment is provided and roof runoff is discharged directly to the detention system. The network is arranged in two main trunk lines: the northern downspouts drain to DT2, and the southern downspouts drain to DT1.

Based on the Client's indication that the roof will be double-pitched with downspouts on both the north and south façades, the current configuration routes a portion of the southern trunk line through the Gantry (generator area). It is recommended to provide a minimum clearance between the two buildings to ensure the roof-drain network is structurally feasible, and locate all inspection manholes outside the Gantry area to prevent any potential flooding within the Gantry, which is the most sensitive structure on the campus.

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Figure 2.5: Plan view of the roof stormwater network — connection to DT1 and DT2 detention tanks

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3. DESCRIPTION OF THE DESIGN DISCHARGES

The following discharges are planned in the design:

- pressure discharge of stormwater into surface water body (**S1**);
- discharge to the public mixed sewer system (**S2**) that receives:
  - first-flush from surfaces subject to Regional Regulation 4/2006 of the Lombardy Region;
  - flows from the domestic wastewater discharges of project building;
  - draining waters from the tank serving the fire-fighting system;
  - condensate water from machinery cooling systems.

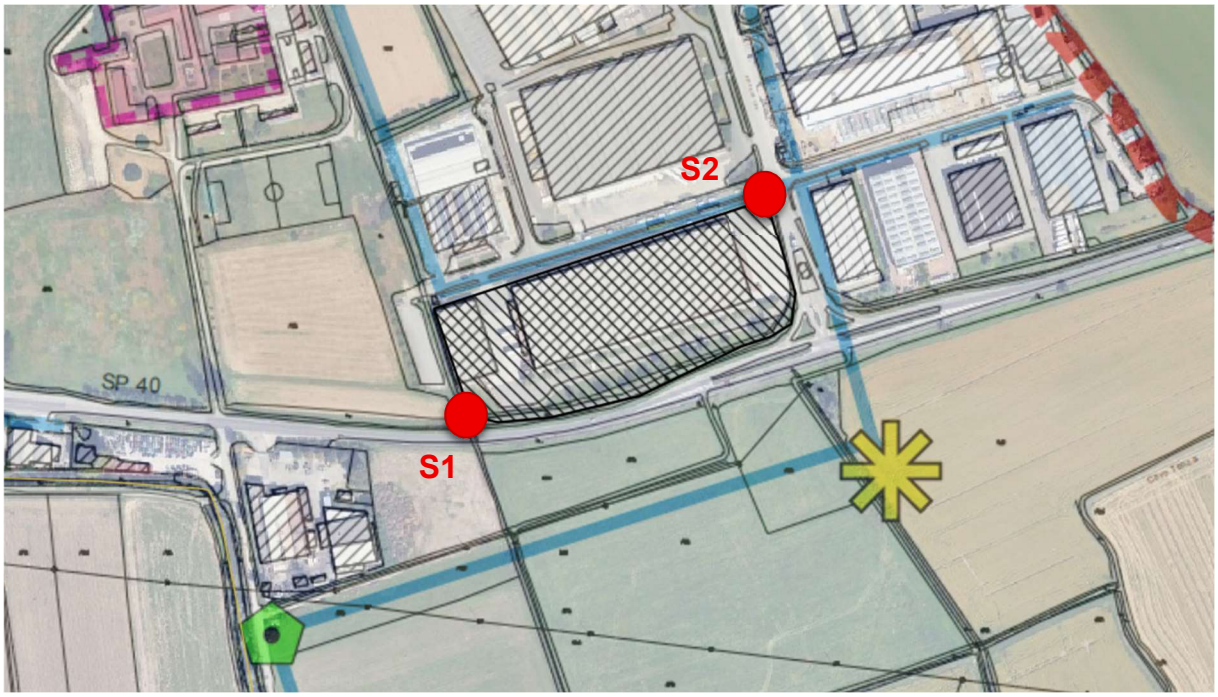

Figure 3.1: Planimetric location of the discharge points (the route of the public sewer in the vicinity of the site is shown in blue)

The coordinates of discharge points, relative to the WGS 84 UTM 32 N (EPSG 32632) design reference system, are reported in the following table.

Table 3.1: Location of design discharges

ID	X [m] WGS 84 UTM 32	Y [m] WGS 84 UTM 32
S1	516600	5019079
S2	516820	5019242

The project's stormwater strategy provides detention via two reinforced-concrete tanks that discharge under pressure to the receiving water body. Green areas are hydraulically independent of the tank system: both the internal green area and the perimeter green strips are to be curbed and slightly depressed relative to adjacent pavements.

Stormwater management for the perimeter green areas will be defined in the next design phase, following topographic surveys to assess potential external inflows toward the site. Nature-Based Solutions (NBS) will be evaluated and adopted as appropriate based on the survey results. The internal green area will manage runoff on site through the storage volume provided by the shallow

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depression and infiltration at the base (DP1). These arrangements are presented as a first-stage hypothesis and will be refined in the subsequent design phase as survey data and detailed design information become available.

The receiving surface water body for the project outfall has been identified as the Roggia Cariola (private drainage network), managed by the Consorzio Naviglio Olona. The ditch is an open channel with natural, revegetated banks; it is currently used for irrigation, and its flow direction is southward.

Downstream of SP40, an approved project provides for the partial relocation/canalization of the Roggia Cariola and for the conveyance of detained flows from another data center planned nearby. Accordingly, the stormwater discharge will be routed from the main detention tank to a single outfall on the Roggia Cariola, via a pressurized pipeline to be finalized in the next design stage (in coordination with the Consorzio and the SP40 works). In all cases, the outfall to the ditch will be equipped with a Clapet (non-return) valve to prevent backflow.

The Consorzio Naviglio Olona has set a maximum allowable discharge to the ditch of 30 l/s in total, which has been apportioned between DT1 and DT2.

The project involves conveying the following discharges to the public sewer system:

- first flush from the generation area (potentially contaminated water subject to Regulation 04/2006) and loading/unloading bay;
- domestic wastewater from project building;
- draining water from the fire-fighting system tank;
- cooling system condensate water.

The domestic wastewater, fire-fighting system draining water, and cooling system condensate are considered assimilated to civil wastewater under Regional Regulation No. 6 of 2019 of the Lombardy Region. These discharges are conveyed to a manhole at the beginning of the mixed sewage system section managed by CAP Holding S.p.A., located on Via Liguria northeast of the project area. This discharge point is identified as **S2** in the project drawings (Figure 3.2). Information about the S2 discharge point was extracted from the network node monograph in the "Acque di Lombardia" WebGIS, which provides a planimetric sketch and tabular data.

Figure 3.2: Planimetric location of discharge point S2 in the WebGIS Acque di Lombardia

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Table 3.2: Monograph of the public sewer network node identified as discharge point S2

ID	OBJECT ID	ITEM TYPE	TYPE OF USE	MANHOLE COVER ELEVATION [m a.s.l.]	MANHOLE BOTTOM ELEVATION [m a.s.l]	X [m] WGS 84 UTM 32	Y [m] WGS 84 UTM 32
810	1002941	Head manhole	Mixed water network	92.54	-	516820	5019242

The project discharge will be carried out in accordance with Annex 2 of the CAP Group's Integrated Water Service Regulation, using a special inspection chamber with a siphon. The final pipeline's last section will have a minimum diameter of DN 200.

Final sizing is deferred to the next design phase.

The wastewater drainage system follows the scheme shown in Figure 3.3. The three network types (civil, industrial, first-flush) are conveyed via separate pipelines to the northern site's security fence. A sampling manhole (PC) is provided for all networks, and for the industrial and stormwater networks, a manhole with a flow meter is included (PM). The three networks merge in a mixing manhole before the ISB manhole (inspection, siphon, collar), and then they reach the discharge S2 of the mixed public sewer in a single wastewater pipe.

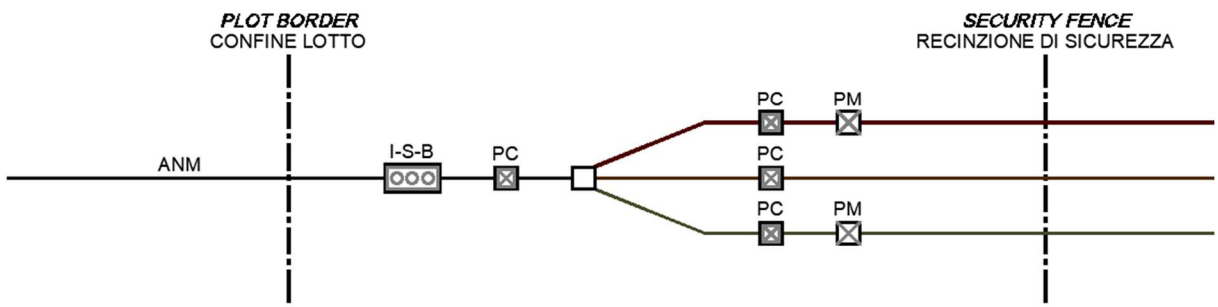

Figure 3.3: Diagram of the wastewater system before discharge into the public sewer

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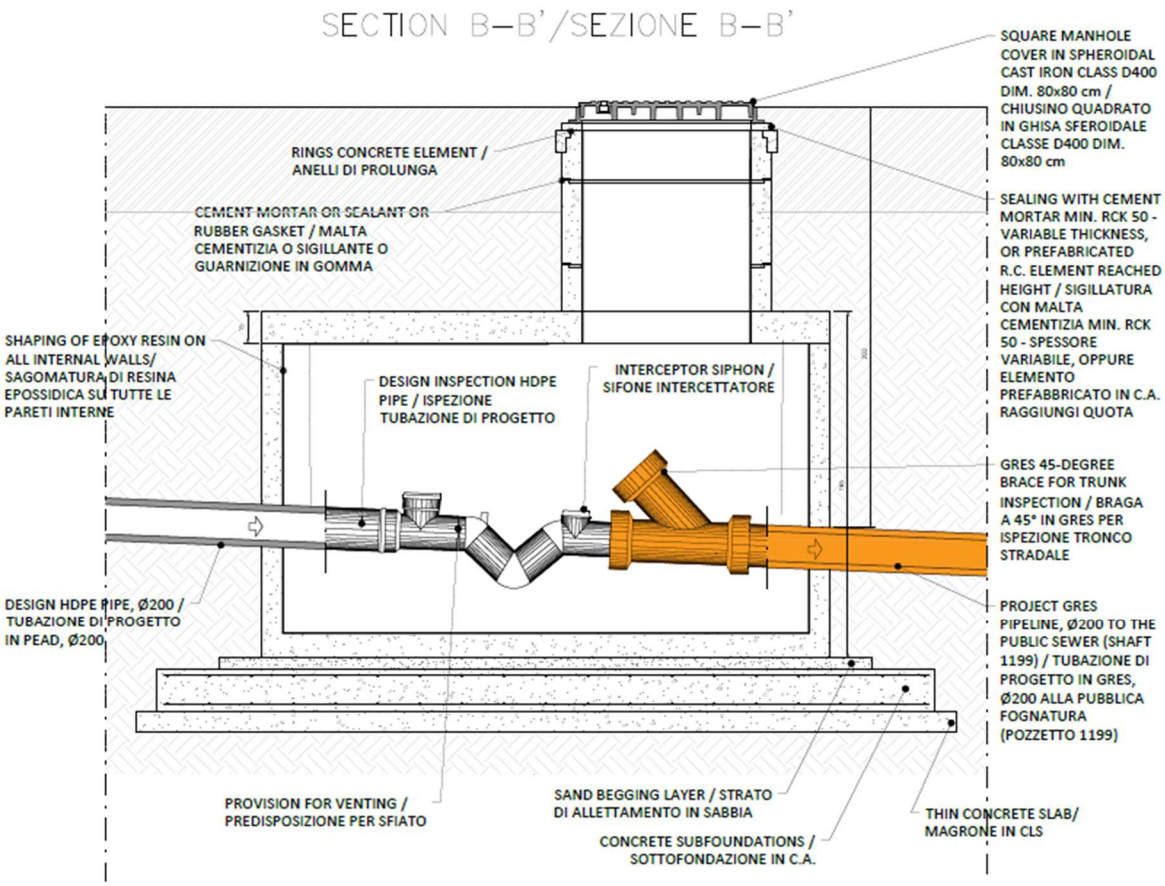

Figure 3.4: Typical inspection chamber type-siphon

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4. STORMWATER DRAINAGE NETWORK

4.1. Hydrological study

4.1.1. Selection of return period

The design rainfall event used in the hydrological calculations and hydraulic simulations is selected based on the concept of return period. The return period of an event refers to the average time between the occurrences of two successive events of equal or greater magnitude than a specified intensity value, or alternatively, the average time over which a specified intensity value is equalled or exceeded at least once.

The probability P_N that an event with a return period T will occur in N years is given by the following formula:

$$PN = 1 - \left(1 - \frac{1}{T}\right)^N$$

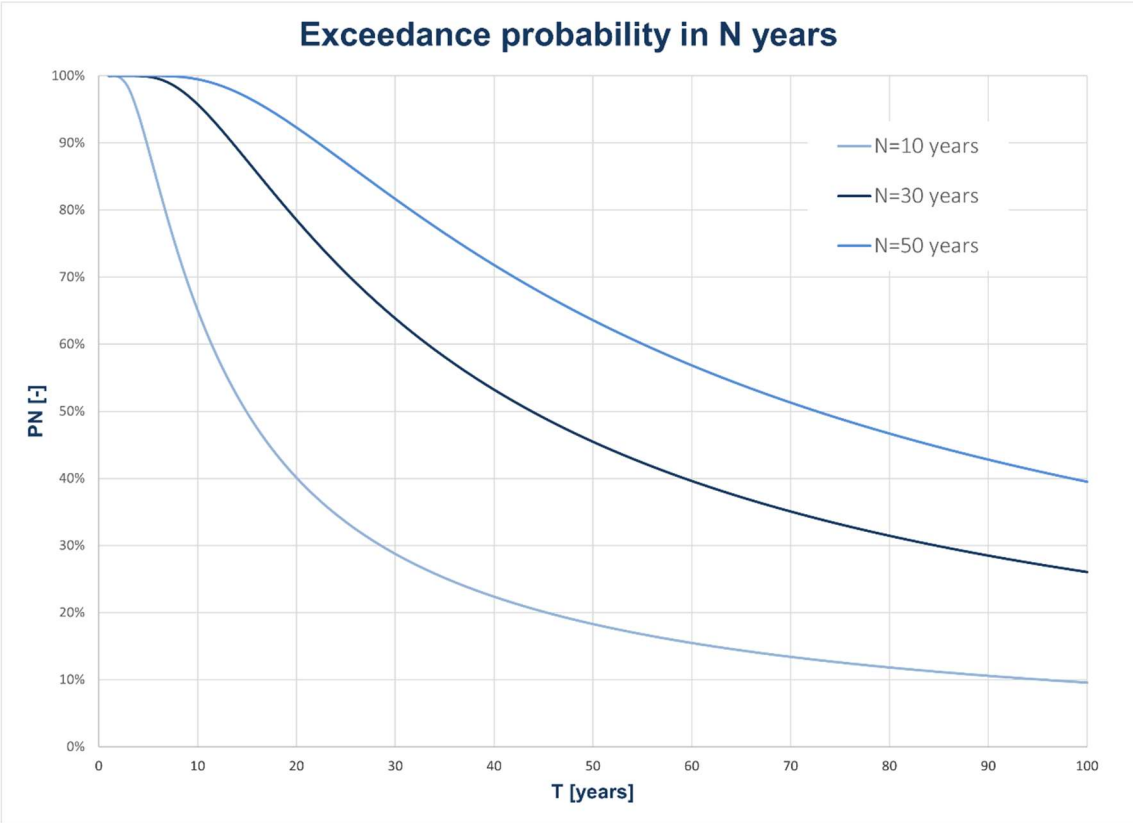

Figure 4.1: Probability of exceedance in N years

Given the sensitivity of the project site to flooding risks, a **50-year** return period was selected as the basis for sizing the street, yard, and roof stormwater collection networks. This design approach ensures the primary drainage systems can adequately manage runoff from high-intensity, low-frequency storm events.

By tailoring the design criteria to the specific functions and criticality of each network, the project team has adopted a prudent and risk-based approach to stormwater infrastructure planning. The 50-year return period for the primary collection systems safeguards against major flooding.

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4.1.2. Determination of DDF curves for various return periods

The Depth-duration-frequency (DDF) curves specify the rainfall depth for a given duration, each referenced to a specified return period.

The formula utilized by ARPA Lombardy is presented below:

$$h(D) = a_1 \cdot w_T \cdot D^n$$

where:

- $h(D)$ represents the rainfall depth for given duration D,
- a_1 and n are characteristic parameters of the rainfall probability curves, which are independent of return period and site-specific,
- w_T is a growth factor calculated based on site-specific parameters of the GEV (Generalized Extreme Value) statistical distribution:

$$w_T = \varepsilon + \frac{\alpha}{k}\left\{1 - \left[ln\left(\frac{T}{T-1}\right)^k\right]\right\}$$

The characteristic parameters for the DDF curves used to determine design precipitation conform to those provided by ARPA Lombardy for all locations within the region. These parameters are accessible via the ARPA Lombardy website, specifically through the link: http://idro.arpalombardia.it/pmapper4.0/map.phtml, covering various return periods and durations ranging from 1 to 24 hours.

For the specific area of interest, the DDF curve parameters are:

$$a_1 = 27.79$$

$$n = 0.2958$$

The GEV statistical distribution parameters are:

$$\alpha = 0.2912$$

$$\varepsilon = 0.8104$$

$$k = -0.069$$

In urban hydrology, where basins are typically small and runoff times are short, the focus is on intense rainfall events lasting less than an hour.

In the absence of alternative data, a value of 0.5 was adopted for the n parameter, as indicated in Annex G of R.R. 7/2017, while keeping the other parameters unchanged.

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Table 4.1: Depth Duration Frequency curves (DDF)

T [years]	2	5	10	50	100	200	500
W_t	0.92	1.27	1.52	2.11	2.39	2.67	3.07
d [hours]			Rainfall depth h [mm]				
0.5	18	25	30	42	47	53	60
1	26	35	42	59	66	74	85
2	31	43	52	72	81	91	105
3	35	49	58	81	92	103	118
4	38	53	64	89	100	112	129
5	41	57	68	95	107	120	137
6	43	60	72	100	113	126	145
7	45	63	75	104	118	132	152
8	47	65	78	109	123	137	158
9	49	68	81	113	127	142	163
10	50	70	83	116	131	147	169
11	52	72	86	119	135	151	173
12	53	74	88	123	138	155	178
13	55	75	90	125	142	159	182
14	56	77	92	128	145	162	186
15	57	79	94	131	148	165	190
16	58	80	96	133	151	169	194
17	59	82	98	136	153	172	197
18	60	83	99	138	156	175	201
19	61	84	101	140	158	177	204
20	62	86	102	143	161	180	207
21	63	87	104	145	163	183	210
22	64	88	105	147	166	185	213
23	65	89	107	149	168	188	216
24	65	90	108	150	170	190	218

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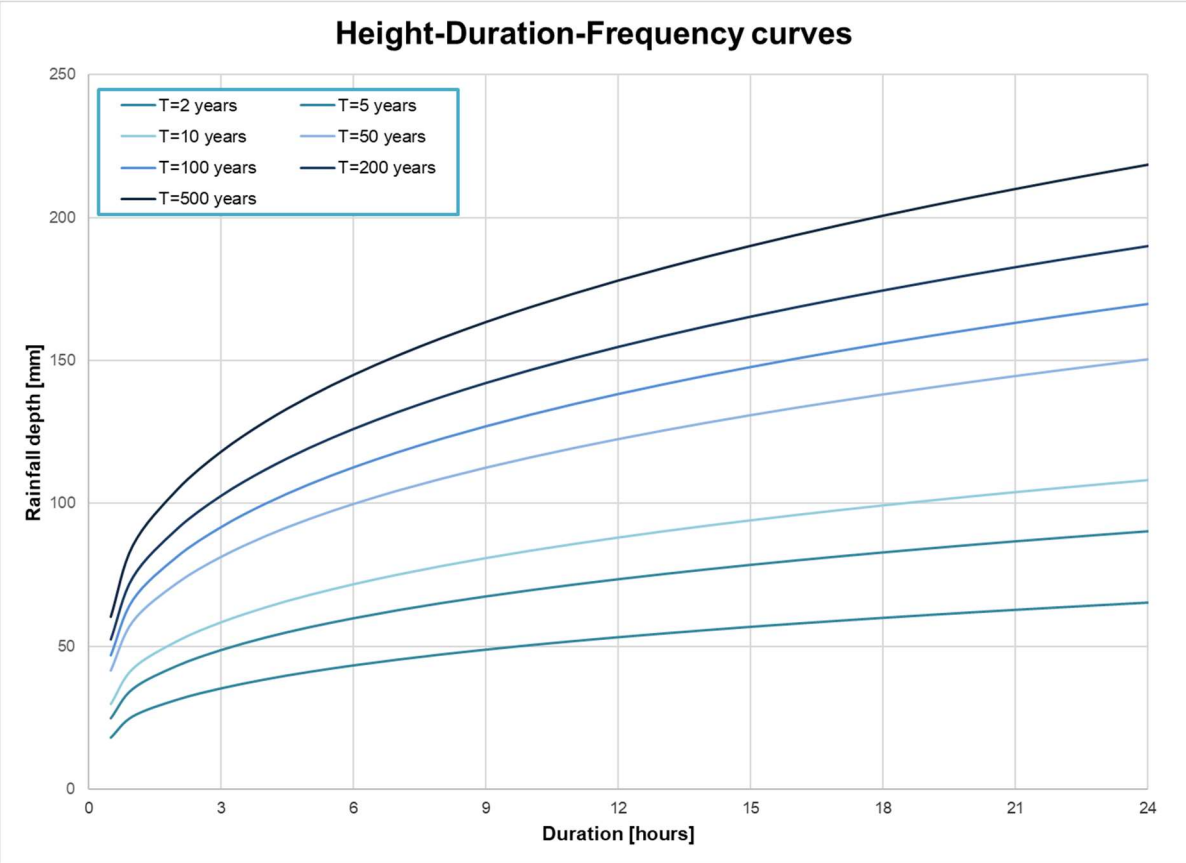

Figure 4.2: Depth Duration Frequency curves (DDF)

Based on the considerations outlined above, the probability curves derived are as follow

- for durations longer than one hour:
  $h(D) = a_1 \, w_T \, D^n$
- for durations shorter than one hour:
  $h(D) = a_1 \, w_T \, D^{0.5}$

4.1.3. Description of design drainage areas and runoff coefficients

To determine the design flow rate, the impermeable drainage area of the site is calculated.

The site area is divided into sub-areas based on their surface cover types; each assigned a specific permeability coefficient according to Regional Regulation 7/2017 and subsequent amendments of the Lombardy Region.

The effective rainfall, which represents the portion of precipitation contributing to the flood wave formation, is calculated using the percentage method. This method assumes a constant percentage of rainfall loss during the event. The runoff coefficient, denoted as φ, characterizes the event by comparing the volume of net precipitation (P_{net}) to total precipitation (P):

$$\varphi = \frac{P_{net}}{P}$$

Net rainfall is obtained by multiplying the synthetic design rainfall by the runoff coefficient φ, assuming uniform infiltration and runoff throughout the event.

The following tables provide a detailed breakdown of the design areas and their associated runoff coefficients.

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Table 4.2: Area values and runoff coefficients for different cover types for the entire site

SURFACE TYPE	AREA [m²]	%	RUNOFF COEFFICIENT [φ]
Building covered area	9917	94	1
Loading bay	454	4	1
Technical room	112	1	1
Asphalted roads & pedestrian walkways	7679	73	1
Detention tanks (DT1, DT2)	707	7	1
Permeable parking area	1067	10	0.7
Photovoltaic panels	157	1	0.7
Detention pond (DP1)	190	2	0.3

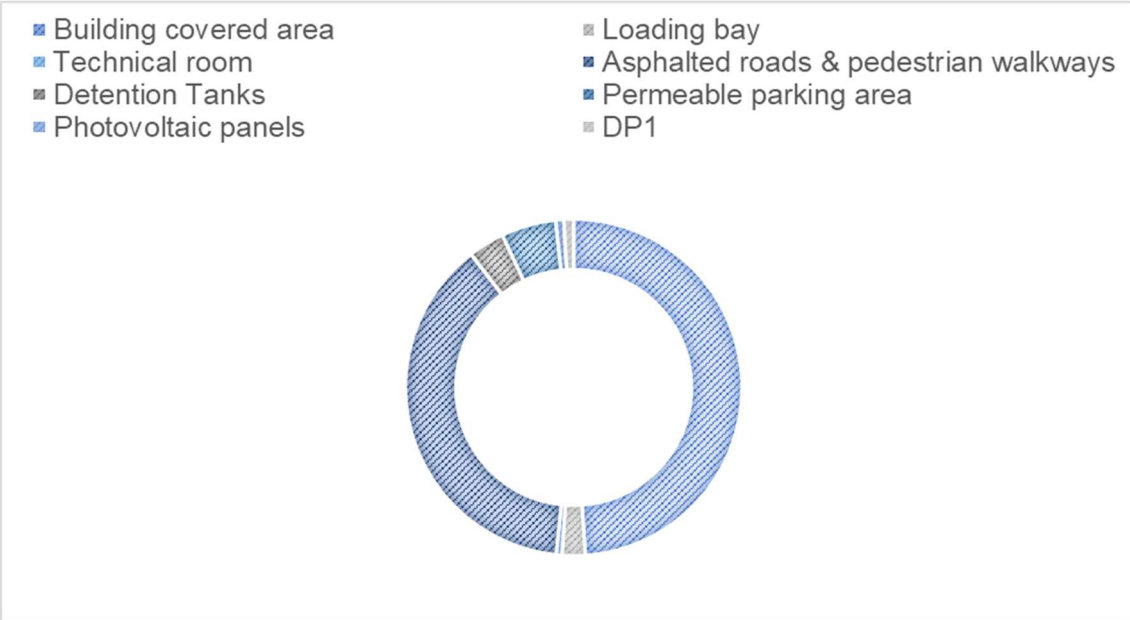

Figure 4.3: Land use distribution

The weighted average runoff coefficient across the entire site is calculated to be 0.98, covering a total impermeable area of 1.98 hectares.

It should be emphasized that the green areas are hydraulically independent of the tank system: the perimeter green strips are to be curbed and slightly depressed relative to adjacent pavements; the runoff of internal green area is managed with a detention basin (DP1).

4.1.4. Rainfall-runoff model

The kinematic model is adopted for defining the draining flow rate, assuming that the precipitation is at constant intensity and the basin time-area curve is linear, the critical duration coincides with the basin runoff time, and the critical flow rate (design flow rate) is given by the expression:

$$Q_{cr}=\varphi\cdot\frac{i(T_r,t_c)\cdot A}{360}$$

Where:

- Q_{cr} = critical net flow rate - [m³/s];
- φ = runoff coefficient by which losses through infiltration and surface detention are considered – [-];
- $i(T_r,t_c)$ = average intensity of precipitation of duration equal to the catchment concentration time (t_c) and having a return period T_r – [mm/h];
- A = catchment surface – [ha].

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The kinematic method is used to define the concentration time, which considers the network entry time and the travel time related to the velocity of the fluid in the pipeline calculated for each of the linear elements of the networks.

4.2. Streets and yards stormwater network

4.2.1. Design Criteria

For collecting stormwater runoff from the yards, a dedicated drainage network was designed capable of gravity conveying the expected rainfall with a return period of **50 years**. The basin **DT1** is designed for west and south network, while the east network will discharge in **DT2**.

The network for collecting water from the yards was designed considering the following assumptions:

- use of corrugated HDPE piping SN 8 kN/m^2;
- the internal roadway will be characterized by single-slope road cross-section of not less than 2%, graded outward toward the site perimeter; the parking areas on the north side of the site will have a transverse slope of not less than 2%, draining toward the internal roadway (Figure 4.4);
- each storm drain will provide a minimum bottom of 30 cm for an initial settling of solids and will be connected to the inspection manholes of the underground network;
- A single row of storm drains will be installed along the curb at the foot of the sidewalks, located at the low points of the cross slope;
- It will be ensured that there is a minimum covering of 70 cm;
- all storm drains and all manholes in the project will have a cover with strength class of D400;
- main pipelines will have a minimum longitudinal slope of not less than 0.3%;
- the ground surface is currently assumed to be flat, and the design has been developed using relative elevations referenced to the existing ground (pending confirmation of absolute site benchmarks and roadway longitudinal profiles);
- the preliminary steady flow sizing of the network assumed a maximum fill rate equal to 75%;

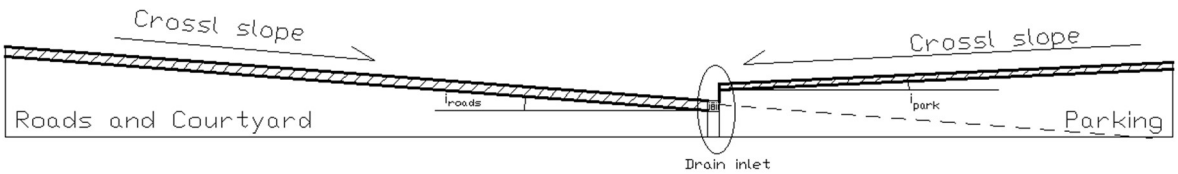

Figure 4.4: Schematic cross-section of the road at a parking area

4.2.2. Definition of design flow rates

A rainfall-runoff model (kinematic or runoff model) was used to define the design flow rates related to the draining areas at the site.

The catchment area affecting the road interception works is approximately 8746 m^2, including roads, parking lots, sidewalks, and yards. This area was divided for sizing into sub-areas of influence related to individual interception works.

The network entry time was assumed to be 5 minutes.

The runoff coefficient is set equal to 1 for impervious areas and 0.7 for semi pervious areas.

A value of Strickler's roughness coefficient was assumed equal to 85 m$^{1/3}$/s.

The return time considered is equal to **50 years** for the sizing of the network and equal to **100 years** for the verification of the safety francs.

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4.2.3. Sizing of storm drains

Stormwater collection is provided through a single row of storm drains positioned adjacent to the sidewalk.

The spacing between the interceptors was sized according to the identified drainage areas and with the aim of limiting the width of flooding on the street during drainage between the ridge and the storm drain. Two successive storm drains are placed around 15 meters apart. The maximum area subtended by each culvert is generally no greater than 300 m^2.

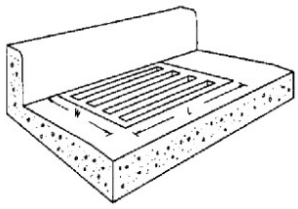

Figure 4.5: Typical storm drains

Chezy's formula was used for sizing the drains and determining their spacing:

$$Q= \frac{C_f}{n} S_x^{5/3} T^{8/3} S_0^{1/2}$$

Where:

- C_f = 0.376;
- n roughness coefficient according to Manning placed equal to 0.017 s/$m^{1/3}$;
- T Maximum width at the top of the wetted section;
- S_x gutter cross slope;
- S_0 longitudinal slope of the road

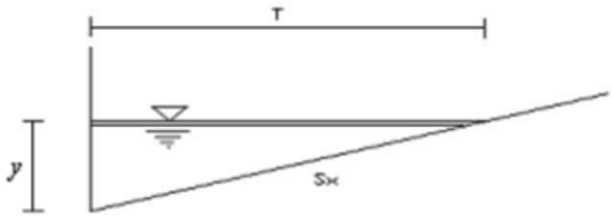

Figure 4.6: Cross-sectional area at the storm drain

The storm drains are planned to be of grid type with internal span equal to 60x60 cm. Each storm drain should provide a minimum bottom of 30 cm for initial settling of solids.

As part of the project, linear drainage grates with concrete channels are planned to collect water from both the loading bay and generator area. In the generator area, a single channel with a bottom slope of 0.4% and a concrete structure with commercial dimensions (from catalog) is planned. Based on the maximum flow rate calculated using the rational method and considering the assumed slope of 0.4% dimensions of the channel are shown in Table 4.3.

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Table 4.3: Internal dimensions of concrete drainage channel - generators' area

UPSTREAM NODE	DOWNSTREAM NODE	SLOPE	DESIGN FLOW RATE	DIMENSION FROM CATALOG
ID	ID	[%]	[m^3/s]	[mm]
SPP.S.1	SPP.S.2	0.4	0.177	500x500Hx1000L (165 units)

For the loading and unloading bay, it is planned to maintain the hydraulic continuity of the drainage system by placing one channel with grates, with commercial dimensions (from catalog) as shown in Table 4.4.

Table 4.4: Internal commercial dimensions of concrete drainage channels – loading bay

DOWNSTREAM NODE	SLOPE	DESIGN FLOW RATE	DIMENSION FROM CATALOG
ID	[%]	[m^3/s]	[mm]
SPP.S.3	0.3	0.025	225x325Hx1000L (16 units)

The channels, equipped with linear grates, are also adopted as the drainage solution for other strategic points such as the site entrance and in other cases where design slopes did not allow drainage along the road or through storm drains.

The discharge from the channels is planned through special elements, which allow either direct discharge from the channel bottom or discharge through a manhole. The geometric parameters and design specifications for the channels in the project are provided in Table 4.5.

A value of Strickler's roughness coefficient for the channel was assumed equal to 70.

Table 4.5: Internal commercial dimensions of concrete drainage channels

DOWNSTREAM NODE	SLOPE	DESIGN FLOW RATE	DIMENSION FROM CATALOG
ID	[%]	[m^3/s]	[mm]
SAO.NW.9.1	0.3	0.012	150x240Hx1000L (8 units)
SAO.NE.8.1	0.3	0.012	150x240Hx1000L (5 units)
SAO.NE.8.2	0.3	0.005	150x240Hx1000L (6 units)
SAO.NE.7.1	0.3	0.004	150x240Hx1000L (6 units)
SAO.NE.11.1	0.3	0.005	150x240Hx1000L (6 units)

The discharge and conveyance capacity of the main pipelines are outlined in the following sections.

4.2.4. Sizing and verifying streets and yards stormwater network

The connection scheme between the storm-drains and the main pipeline foresees the positioning of inspection manholes at a maximum distance of 15 meters between two successive manholes since the manholes are directly connected to the gully inlets. The inspection/connection manholes will be in reinforced concrete complete with covers with D400 roadway resistance class.

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For connections between drains and between drains and inspection manholes, pipes of diameter DN250 with a slope of 1%. At manholes, the minimum pipe overlay is 0.50 m.

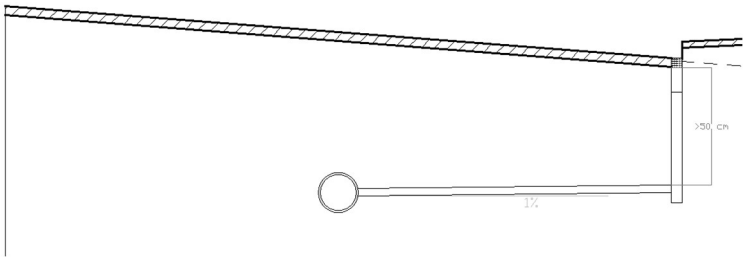

Figure 4.7: Schematic cross-section of the gully–manhole connection

The verification of the single gully was carried out assuming a drainage area of 300 m² and the following design parameters:

Table 4.6: Verifying the connection diagram between storm-drains and manhole

PARAMETER	VALUE
RETURN PERIOD	20 years
INTENSITY RAINFALL	170 mm/h
WIDTH OF ROADWAY DRAINING TO CURB	10 m
CROSS SLOPE	2%
ALLOWABLE TOP WIDTH OF DRAINING AGAINST CURB	1.5 m
LONGITUDINAL SLOPE	0.1%
GUTTER WIDTH	60 cm
MAX FLOW RATE (FROM RATIONAL METHOD FOR PICK EVENT)	14 l/s

Considering these parameters, the maximum spacing of the gullies is 15 m and the minimum inflow perimeter is 1.0 m. Therefore, spacings of 15 m have been adopted, with gullies measuring 60 cm per side, thus providing an inflow perimeter of 1.8 m.

The maximum flooding depth above the gully is 5 cm, with a maximum flooding distance from the sidewalk of 2.6 m. The outflow occurs over a weir, without any conditions of submergence or gully overflush.

The underground network includes a total of three main pipeline that will have increasing diameters from upstream to downstream as the inputs increase. The minimum internal diameter planned is 250 mm and the maximum is 701 mm, which will be used in the final sections before the water is collected at the DT1. For DT2 minimum diameter upstream is a 250 mm and the maximum is a 500 mm.

- One pipeline will convey the water collected from the drains of streets and yards to detention tank DT1;
- One pipeline will convey the water collected from the drains of streets and yards to detention tank DT2;
- One pipeline will be dedicated to the drainage grates of the loading bay and the area destined for the generators (with head sump downstream of the pre-treatments in compliance with R.R. 04/2006 of the Lombardy Region);

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- One pipeline to convey the second flush from the drainage of the areas subjected to R.R. 04/2006 to the detention tank DT2.

Although they are not strictly subject to R.R. 04/2006, the water from roads and yards will undergo slight continuous pretreatment before reaching the tank.

The geometric and fluid-dynamic data of the planned backbones are illustrated below.

Table 4.7: Hydrological parameters for sizing streets and yards stormwater management network SAO.NW

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SAO.NW.1	SAO.NW.2	5.3	70	1.00	198	4
SAO.NW.2	SAO.NW.3	5.5	467	0.84	193	21
SAO.NW.3	SAO.NW.4	5.7	864	0.84	191	38
SAO.NW.4	SAO.NW.5	5.8	1030	1.00	188	54
SAO.NW.5	SAO.NW.6	6.0	1295	0.84	186	56
SAO.NW.6	SAO.NW.7	6.1	1546	0.84	185	67
SAO.NW.7	SAO.NW.8	6.2	1711	1.00	183	87
SAO.NW.8	SAO.NW.9	6.3	1876	1.00	182	95
SAO.NW.9	SAO.NW.10	6.4	2093	1.00	180	105
SAO.NW.10	SAO.NW.11	6.5	2237	1.00	179	111
SAO.NW.11	SAO.NW.12	6.6	2396	1.00	177	118
SAO.NW.12	SAO.NW.13	6.8	2517	1.00	175	122
SAO.NW.13	SAO.NW.14	6.9	2723	1.00	173	131
SAO.NW.14	SAO.NW.15	7.0	2996	1.00	172	143
SAO.NW.15	SAO.NW.16	7.1	3178	1.00	171	151
SAO.NW.16	SAO.OUT.W.1	7.2	3299	1.00	170	156

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Table 4.8: Verifying of streets and yards stormwater management network SAO.NW

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAO.NW.1	SAO.NW.2	0.70	20.00	218	0.76	23
SAO.NW.2	SAO.NW.3	0.76	20.00	250	0.83	33
SAO.NW.3	SAO.NW.4	0.82	11.70	300	0.94	53
SAO.NW.4	SAO.NW.5	0.86	12.70	347	1.03	79
SAO.NW.5	SAO.NW.6	0.89	12.30	347	1.03	79
SAO.NW.6	SAO.NW.7	0.93	11.00	400	1.14	115
SAO.NW.7	SAO.NW.8	0.96	11.00	400	1.14	115
SAO.NW.8	SAO.NW.9	1.00	10.00	400	1.14	115
SAO.NW.9	SAO.NW.10	1.03	9.00	400	1.14	115
SAO.NW.10	SAO.NW.11	1.05	13.00	400	1.14	115
SAO.NW.11	SAO.NW.12	1.09	11.50	433	1.20	142
SAO.NW.12	SAO.NW.13	1.13	16.50	433	1.20	142
SAO.NW.13	SAO.NW.14	1.18	16.50	433	1.20	142
SAO.NW.14	SAO.NW.15	1.23	11.50	500	1.32	208
SAO.NW.15	SAO.NW.16	1.26	11.70	500	1.32	208
SAO.NW.16	SAO.OUT.W.1	1.30	8.00	500	1.32	208

Table 4.9: Hydrological parameters for sizing streets and yards stormwater management network SAO.SW

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SAO.SW.1	SAO.SW.2	5.2	382	0.93	199	20
SAO.SW.2	SAO.SW.3	5.4	671	0.98	196	36
SAO.SW.3	SAO.SW.4	5.5	964	1.00	193	52
SAO.SW.4	SAO.SW.5	5.7	1257	1.00	191	67
SAO.SW.5	SAO.SW.6	6.0	1407	1.00	186	73
SAO.SW.6	SAO.SW.7	6.0	484	1.00	186	25
SAO.SW.6	SAO.SW.8	6.0	1891	1.00	185	97
SAO.SW.8	SAO.SW.9	6.2	2030	1.00	184	103
SAO.SW.9	SAO.SW.10	6.3	2179	1.00	181	110
SAO.SW.10	SAO.SW.11	6.4	2498	1.00	180	125
SAO.SW.11	SAO.SW.12	6.6	2813	1.00	178	139
SAO.SW.12	SAO.OUT.W.1	6.7	3085	0.94	176	142

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Table 4.10: Verifying of streets and yards stormwater management network SAO.SW

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAO.SW.1	SAO.SW.2	0.70	14.00	218	0.76	23
SAO.SW.2	SAO.SW.3	0.74	14.00	300	0.94	53
SAO.SW.3	SAO.SW.4	0.78	14.00	300	0.94	53
SAO.SW.4	SAO.SW.5	0.83	12.30	347	1.03	79
SAO.SW.5	SAO.SW.6	0.86	27.00	347	1.03	79
SAO.SW.6	SAO.SW.7	0.70	16.00	400	1.14	115
SAO.SW.6	SAO.SW.8	0.94	7.70	400	1.14	115
SAO.SW.8	SAO.SW.9	0.97	12.00	400	1.14	115
SAO.SW.9	SAO.SW.10	1.00	14.00	400	1.14	115
SAO.SW.10	SAO.SW.11	1.05	14.00	433	1.20	142
SAO.SW.11	SAO.SW.12	1.09	14.00	433	1.20	142
SAO.SW.12	SAO.OUT.W.1	1.13	12.60	433	1.20	142

Table 4.11: Hydrological parameters for sizing streets and yards stormwater management network SAO.NE

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SAO.NE.1	SAO.NE.2	5.2	70	1.00	199	4
SAO.NE.2	SAO.NE.3	5.4	376	0.86	196	18
SAO.NE.3	SAO.NE.4	5.6	633	0.87	193	29
SAO.NE.4	SAO.NE.5	5.7	843	0.92	191	41
SAO.NE.5	SAO.NE.6	5.8	1055	0.92	189	51
SAO.NE.6	SAO.NE.7	6.0	1226	1.00	186	63
SAO.NE.7	SAO.NE.8	6.0	304	1.00	186	16
SAO.NE.7	SAO.NE.9	6.0	1610	1.00	185	83
SAO.NE.9	SAO.NE.10	6.0	117	1.00	185	6
SAO.NE.9	SAO.NE.11	6.1	1802	1.00	184	92
SAO.NE.11	SAO.NE.12	6.2	1900	1.00	182	96
SAO.NE.12	SAO.NE.13	6.3	2057	1.00	181	104
SAO.NE.13	SAO.NE.14	6.3	153	1.00	181	8
SAO.NE.13	SAO.NE.15	6.3	2210	1.00	181	111
SAO.NE.15	SAO.NE.16	6.5	2329	1.00	179	116
SAO.NE.16	SAO.NE.17	6.6	2451	1.00	177	121
SAO.NE.17	SAO.NE.18	6.8	2569	1.00	175	125
SAO.NE.18	SAO.OUT.E.1	6.8	2864	1.00	174	139

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Table 4.12: Verifying of streets and yards stormwater management network SAO.NE

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAO.NE.1	SAO.NE.2	0.70	15.00	218	0.76	23
SAO.NE.2	SAO.NE.3	0.75	13.00	218	0.76	23
SAO.NE.3	SAO.NE.4	0.78	10.30	250	0.83	33
SAO.NE.4	SAO.NE.5	0.81	10.00	300	0.94	53
SAO.NE.5	SAO.NE.6	0.84	9.70	300	0.94	53
SAO.NE.6	SAO.NE.7	0.87	17.00	347	1.03	79
SAO.NE.7	SAO.NE.8	0.70	11.40	218	0.76	23
SAO.NE.7	SAO.NE.9	0.93	7.00	400	1.14	115
SAO.NE.9	SAO.NE.10	0.70	6.70	218	0.76	23
SAO.NE.9	SAO.NE.11	0.95	7.00	400	1.14	115
SAO.NE.11	SAO.NE.12	0.97	12.60	400	1.14	115
SAO.NE.12	SAO.NE.13	1.00	7.00	400	1.14	115
SAO.NE.13	SAO.NE.14	0.70	3.70	218	0.76	23
SAO.NE.13	SAO.NE.15	1.03	5.40	400	1.14	115
SAO.NE.15	SAO.NE.16	1.04	13.60	433	1.20	142
SAO.NE.16	SAO.NE.17	1.08	13.40	433	1.20	142
SAO.NE.17	SAO.NE.18	1.12	17.30	433	1.20	142
SAO.NE.18	SAO.OUT.E.1	1.17	6.00	433	1.20	142

Table 4.13: Hydrological parameters for sizing streets and yards stormwater management network SAO.OUT.W

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SAO.OUT.W.1	SAO.OUT.W.2	7.2	6642	1.00	170	313
SAO.OUT.W.2	SAO.OUT.W.3	7.2	6642	1.00	170	313
SAO.OUT.W.3	SAO.OUT.W.4	7.3	6642	1.00	168	311
SAO.OUT.W.4	DT1	7.4	6642	1.00	168	309

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Table 4.14: Verifying of streets and yards stormwater management network SAO.OUT.W

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAO.OUT.W.1	SAO.OUT.W.2	1.32	4.00	600	1.49	339
SAO.OUT.W.2	SAO.OUT.W.3	1.33	1.00	600	1.49	339
SAO.OUT.W.3	SAO.OUT.W.4	1.33	12.60	600	1.49	339
SAO.OUT.W.4	DT1	1.37	8.50	600	1.49	339

Table 4.15: Hydrological parameters for sizing pipeline street and courtyard SAO.OUT.E

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SAO.OUT.E.1	SAO.OUT.E.2	6.85	2864	1.00	174	138
SAO.OUT.E.2	SAO.OUT.E.3	6.86	2864	1.00	174	138
SAO.OUT.E.3	SAO.OUT.E.4	6.99	2864	1.00	172	137
SAO.OUT.E.4	DT2	7.02	2864	1.00	172	137

Table 4.16: Verifying of streets and yards stormwater management network SAO.OUT.E

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAO.OUT.E.1	SAO.OUT.E.2	1.19	4.60	433	1.20	142
SAO.OUT.E.2	SAO.OUT.E.3	1.21	1.00	433	1.20	142
SAO.OUT.E.3	SAO.OUT.E.4	1.21	13.30	433	1.20	142
SAO.OUT.E.4	DT2	1.25	3.70	433	1.20	142

Table 4.17: Hydrological parameters for potentially oil-contaminated network sizing SPP.S

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m^2]	[-]	[mm/h]	[l/s]
SPP.S.2	SPP.S.5	5.25	3627	1	199	200
SPP.S.3	SPP.S.4	5.13	454	1	201	25
SPP.S.4	SPP.S.5	5.25	454	1	199	25
SPP.S.5	FFT	5.39	4081	1	196	222
FFT	SAO.OUT.E.1	5.47	4081	1	195	221

Table 4.18: Verifying of the potentially oil-contaminated network for roof SPP.S

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY

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US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SPP.S.2	SPP.S.5	0.40	2.00	250	0.83	218
SPP.S.3	SPP.S.4	0.70	10.00	250	0.83	33
SPP.S.4	SPP.S.5	0.73	8.60	250	0.83	33
SPP.S.5	FFT	0.76	18.00	546	1.40	264
FFT	SAO.OUT.E.1	0.81	9.30	546	1.40	264

4.2.5. Inspection/connection manholes of street and yard stormwater network

Connecting manhole in reinforced concrete complete with cast-iron manhole covers are provided.

The dimensions of the inner base of the inspection and connection manholes along the network are chosen depending on the pipe size, according to the following criteria:

- 0.5 x 0.5 m if the diameter of the inlet/outlet pipes exceeds 200 mm and does not exceed 300 mm;
- 0.6 x 0,6 m if the diameter of the inlet/outlet pipework is greater than 300 mm and does not exceed 350 mm;
- 0.8 x 0.8 m if the diameter of the inlet/outlet pipes is greater than 350 mm and does not exceed 500 mm;
- 1 x 1 m if the diameter of the inlet/outlet pipes is greater than 500 mm and not exceeding 700 mm;
- 1.2 x 1.2 m if the diameter of the inlet/outlet pipework is greater than 700 mm and not exceeding 900 mm.

A classification of manholes in relation to internal dimensions is presented below (Table 4.19). In Table 4.20 and Table 4.21, the type is matched to each manhole in the network.

Table 4.19: Manhole types with relative internal dimensions

MANHOLE TYPE	INNER DIMENSIONS
ID	[cm]
RW.IM.01	50x50
RW.IM.02	60x60
RW.IM.03	80x80
RW.IM.04	100x100
RW.IM.05	120x120

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Table 4.20: Manhole types of the streets and yards stormwater network SAO

MANHOLE	TYPE	MANHOLE	TYPE	MANHOLE	TYPE
SAO.NW.1	RW.IM0.1	SAO.SW.1	RW.IM0.1	SAO.NE.5	RW.IM0.3
SAO.NW.2	RW.IM0.1	SAO.SW.2	RW.IM0.3	SAO.NE.6	RW.IM0.3
SAO.NW.3	RW.IM0.3	SAO.SW.3	RW.IM0.3	SAO.NE.7	RW.IM0.3
SAO.NW.4	RW.IM0.3	SAO.SW.4	RW.IM0.3	SAO.NE.8	RW.IM0.1
SAO.NW.5	RW.IM0.3	SAO.SW.5	RW.IM0.3	SAO.NE.9	RW.IM0.3
SAO.NW.6	RW.IM0.3	SAO.SW.6	RW.IM0.3	SAO.NE.10	RW.IM0.1
SAO.NW.7	RW.IM0.3	SAO.SW.7	RW.IM0.3	SAO.NE.11	RW.IM0.3
SAO.NW.8	RW.IM0.3	SAO.SW.8	RW.IM0.3	SAO.NE.12	RW.IM0.3
SAO.NW.9	RW.IM0.3	SAO.SW.9	RW.IM0.3	SAO.NE.13	RW.IM0.3
SAO.NW.10	RW.IM0.3	SAO.SW.10	RW.IM0.4	SAO.NE.14	RW.IM0.1
SAO.NW.11	RW.IM0.4	SAO.SW.11	RW.IM0.4	SAO.NE.15	RW.IM0.4
SAO.NW.12	RW.IM0.4	SAO.SW.12	RW.IM0.4	SAO.NE.16	RW.IM0.4
SAO.NW.13	RW.IM0.4	SAO.NE.1	RW.IM0.1	SAO.NE.17	RW.IM0.4
SAO.NW.14	RW.IM0.4	SAO.NE.2	RW.IM0.1	SAO.NE.18	RW.IM0.4
SAO.NW.15	RW.IM0.4	SAO.NE.3	RW.IM0.1	SAO.OUT.W.1	RW.IM0.5
SAO.NW.16	RW.IM0.4	SAO.NE.4	RW.IM0.3	SAO.OUT.E.1	RW.IM0.4

Table 4.21: Manhole types of the potentially oil-contaminated network for roof SPP.S

MANHOLE	TYPE
SPP.S.2	RW.IM0.1
SPP.S.3	RW.IM0.1
SPP.S.4	RW.IM0.1
SPP.S.5	RW.IM0.4
FFT	RW.IM0.4

4.2.6. Pre-treatment works for areas subject to R.R. 04/2006

The rainwater flowing over the areas that house the fuel tanks are subject to the application of R.R. 04/2006 and therefore require appropriate sand and oil separation treatment to ensure compliance with the limits established by the current regulations governing discharges (Legislative Decree no. 152/2006 and subsequent amendments and supplements). Although the loading/unloading bay is not subject to the requirements of Regional Regulation 4/2006 of the Lombardy Region, it has been included in the potentially oil-contaminated drainage network due to site topography. These areas are located in the southern part of the site.

Based on the information provided by the Client, no waste storage areas or other areas potentially subject to first-flush requirements have been identified.

The potentially oily runoff is intercepted by two linear drainage grates – one serving the generator area and one serving the loading bay – and is conveyed to a dedicated drainage network. This network discharges to a spillway manhole that separates the first-flush from the remainder of the rainfall.

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The total surface areas considered for the sizing of the first rainwater tank and related treatment system are summarized in the table below.

Table 4.22 Surfaces subject to the application of R.R. 04/2006 and loading/unloading bay

SURFACE ID	[m²]
Generator and transformers area	3627
Loading bay	454
Total	4081

The project provides for the construction of one first-flus tank built in the southern area of the site, adjacent to the gantry.

The first-flush tank have been sized by considering the first 5 mm of gross rainfall depth on the surface of the gantry and loading bay:

$$W_{pp1,2} = 5\ mm \cdot (S_{gantry} + S_{loading\ bay}) = 0{,}005\ m \cdot (3627\ m^2 + 454\ \ m^2) = 21\ m^3$$

The treatment system will consist of pre-cast reinforced concrete elements with the usable volume of up to 21 m^3.

Upstream of the tank there will be a spillway manhole to separate the stormwater from the first-flush. Downstream of the tank there will be a de-oiling section. The first-flush tank will be equipped with a float-type backflow preventer valve to ensure proper separation of the first-flush.

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4.2.7. Pre-treatment works for areas not subject to application of R.R. 04/2006

The runoff from streets and yards, although not subject to Regional Regulation 04/2006, will still be treated prior to discharge into the three detention tanks in design (DT1 and DT2).

The two treatments will take immediately upstream of each detention tank at the end of the two pipelines:

- SAO.NW (**DT1**);
- SAO.NE (**DT2**).

The surface area of roads and yards of interest for each treatment system is reported in the following table.

Table 4.23: Surfaces not subject to the application of R.R. 04/2006 treated prior to discharge

SURFACE ID	m^2
Streets and yards SAO.NW (DT1)	5159
Streets and yards SAO.WS (DT2)	2520
Total	7679

The volume of the work is sized by considering the treatment of the first 5 mm of precipitation over the entire surface area in 15 minutes.

The pre-treatment system consists of a prefabricated element that consists of two compartments in series, the first of which performs the function of a desander while in the second compartment the gravimetric flotation of the oils takes place.

The design volume of each treatment is reported below:

Table 4.24: First Flush Volumes of areas not subject to the application of R.R. 04/2006 treated prior to discharge

SURFACE ID	m^3	RESIDENCE TIME [min]	FLOW RATE [l/s]
Streets and yards SAO.NW (DT1)	26	5	90
Streets and yard SAO.WS (DT2)	13	5	45

A spillway manhole is placed upstream of each plant to allow discharge into the detention tank without backflow to the network during peak events.

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4.3. Roof stormwater network

4.3.1. Design Criteria

The roof stormwater collection network was sized using the following assumptions:

- use of HDPE corrugated pipes SN 8 kN/m^2;
- all manholes in the project will have a strength class of D400;
- main pipelines will have a minimum longitudinal slope of not less than 0.3%;
- It will be ensured that there is a minimum covering of 70 cm.
- the preliminary steady flow sizing of the network assumed a maximum fill rate equal to 75%.

The positioning of the downspout manholes has been considered in the facade drainage design and all the calculations set out below are valid starting from the manhole at the base of the downspout and continuing downstream.

The entire hydrological flow rate of the draining roof portion at the downpipe is assumed to be the hydraulic flow rate discharged directly to the sump at the base of the downpipe.

4.3.2. Definition of design flow rates

A rainfall-runoff model (kinematic or runoff model) was used to define the design flow rates for the draining areas on the roofs.

The total draining area is approximately 9917 m^2, including the roofs of the data hall and office; this area was subdivided for dimensioning purposes into sub-areas of influence relating to individual downspout.

The inlet time was assumed to be 5 minutes based on the size of the drainage areas.

The runoff coefficient was set equal to 1.

The Strickler's roughness coefficient value was assumed equal to 85 m$^{1/3}$/s.

The return period considered is equal to **50 years** for the sizing of the network and equal to **100 years** for the verifying of the safety freeboard.

4.3.3. Roof stormwater network sizing and verifying

Pending receipt of the roof layout and exact downspout locations, they have been assumed along both the north and south façades, consistent with a dual-pitch roof with a central ridge running along the building's length (half of the roof draining north and half south).

The underground network includes two main pipelines that collect the downspouts: a North trunk (SAB.NE) running along the north side and a South trunk (SAB.SW) running principally along the south side of the main building. The flows are conveyed into two final outflow headers, which reaches the detention tanks: the North trunk discharges into **DT2**, while the South trunk discharges to **DT1**.

The pipelines will have increasing diameters from upstream to downstream as the inflow increases. The minimum planned internal diameter is 284 mm, and the maximum is 565 mm used in the final sections before the water is collected at the detention tanks.

A coordination note applies to the South trunk, which runs between the main building and the generator gantry: adequate horizontal clearance must be ensured to allow installation, access, and maintenance of the pipeline.

The geometric and hydrologic and hydraulic calculation data of the two planned pipelines are illustrated below.

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Table 4.25: Hydrological parameters for roof stormwater network sizing (SAB.NE)

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m²]	[-]	[mm/h]	[l/s]
SAB.NE.1	SAB.NE.2	5.47	506	1.00	195	27
SAB.NE.2	SAB.NE.3	5.84	1034	1.00	188	54
SAB.NE.3	SAB.NE.4	6.17	1562	1.00	183	79
SAB.NE.4	SAB.NE.5	6.51	2090	1.00	178	104
SAB.NE.5	SAB.NE.6	6.83	2618	1.00	174	127
SAB.NE.6	SAB.NE.7	6.84	3146	1.00	174	152
SAB.NE.7	SAB.NE.8	6.99	3146	1.00	172	150
SAB.NE.8	SAB.NE.9	7.16	3146	1.00	170	149
SAB.NE.9	SAB.NE.10	7.25	3146	1.00	169	148
SAB.NE.10	SAB.NE.11	7.34	3146	1.00	168	147
SAB.NE.11	SAB.NE.12	7.51	3146	1.00	166	145
SAB.NE.12	SAB.NE.13	7.69	3146	1.00	164	143
SAB.NE.13	DT2	7.74	3146	1.00	164	143

Table 4.26: Verifying of roof stormwater network (SAB.NE)

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAB.NE.1	SAB.NE.2	0.70	35.00	250	0.83	33
SAB.NE.2	SAB.NE.3	0.80	34.60	347	1.03	79
SAB.NE.3	SAB.NE.4	0.91	34.30	400	1.14	115
SAB.NE.4	SAB.NE.5	1.01	34.50	400	1.14	115
SAB.NE.5	SAB.NE.6	1.12	34.30	433	1.20	142
SAB.NE.6	SAB.NE.7	1.22	1.50	500	1.32	208
SAB.NE.7	SAB.NE.8	1.22	17.50	500	1.32	208
SAB.NE.8	SAB.NE.9	1.28	20.00	500	1.32	208
SAB.NE.9	SAB.NE.10	1.34	11.30	500	1.32	208
SAB.NE.10	SAB.NE.11	1.37	10.70	500	1.32	208
SAB.NE.11	SAB.NE.12	1.40	20.00	500	1.32	208
SAB.NE.12	SAB.NE.13	1.46	21.30	500	1.32	208
SAB.NE.13	DT2	1.53	5.50	500	1.32	208

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Table 4.27: Hydrological parameters for roof stormwater network sizing (SAB.SW)

NODE ID	NODE ID	TIME OF CONC.	DRAINAGE AREA	RUNOFF COEFF.	DESIGN RAINFALL INTENSITY	DESIGN FLOW RATE
US	DS	[min]	[m²]	[-]	[mm/h]	[l/s]
SAB.SW.1	SAB.SW.2	5.45	506	1.00	195	27
SAB.SW.2	SAB.SW.3	5.82	1034	1.00	189	54
SAB.SW.3	SAB.SW.4	6.15	1562	1.00	183	80
SAB.SW.4	SAB.SW.5	6.49	2090	1.00	179	104
SAB.SW.5	SAB.SW.6	6.80	2618	1.00	174	127
SAB.SW.6	SAB.SW.7	6.89	3146	1.00	173	151
SAB.SW.7	SAB.SW.8	6.95	3146	1.00	173	151
SAB.SW.8	SAB.SW.9	7.05	3146	1.00	171	150
SAB.SW.9	SAB.SW.10	7.30	3146	1.00	168	147
SAB.SW.10	DT1	7.36	3146	1.00	168	147

Table 4.28: Verifying of roof stormwater network (SAB.SW)

NODE ID	NODE ID	OVERLAY	LENGTH	INNER DIAMETER	VELOCITY	DRAIN CAPACITY
US	DS	US [m]	[m]	DI [mm]	[m/s]	[l/s]
SAB.SW.1	SAB.SW.2	0.70	33.40	250	0.83	33
SAB.SW.2	SAB.SW.3	0.80	34.40	347	1.03	79
SAB.SW.3	SAB.SW.4	0.90	34.40	400	1.14	115
SAB.SW.4	SAB.SW.5	1.01	34.30	400	1.14	115
SAB.SW.5	SAB.SW.6	1.11	34.20	433	1.20	142
SAB.SW.6	SAB.SW.7	1.21	10.40	500	1.32	208
SAB.SW.7	SAB.SW.8	1.24	6.50	500	1.32	208
SAB.SW.8	SAB.SW.9	1.26	12.00	500	1.32	208
SAB.SW.9	SAB.SW.10	1.30	29.50	500	1.32	208
SAB.SW.10	DT1	1.39	8.00	500	1.32	208

4.3.4. Inspection/connection manholes of roof stormwater network

Connecting manhole in reinforced concrete complete with cast-iron manhole covers are provided.

The dimensions of the inner base of the inspection and connection manholes along the network are chosen depending on the pipe size, according to the following criteria:

- 0.5 x 0.5 m if the diameter of the inlet/outlet pipes exceeds 200 mm and does not exceed 300 mm;
- 0.6 x 0.6 m if the diameter of the inlet/outlet pipework is greater than 300 mm and does not exceed 350 mm;
- 0.8 x 0.8 m if the diameter of the inlet/outlet pipes is greater than 350 mm and does not exceed 500 mm;
- 1 x 1 m if the diameter of the inlet/outlet pipes is greater than 500 mm and not exceeding 700 mm;

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- 1.2 x 1.2 m if the diameter of the inlet/outlet pipework is greater than 700 mm and not exceeding 900 mm.

A classification of manholes in relation to internal dimensions is presented below (Table 4.20). In Table 4.21, the type is matched to each manhole in the network.

Table 4.29: Manhole types with relative internal dimensions in roof stormwater network

MANHOLE TYPE	INNER DIMENSIONS
ID	[cm]
RW.IM.01	50x50
RW.IM.02	60x60
RW.IM.03	80x80
RW.IM.04	100x100
RW.IM.05	120x120

Table 4.30: Manhole types of the roof stormwater network

MANHOLE	TYPE	MANHOLE	TYPE
ID	ID	ID	ID
SAB.NE.1	RW.IM0.1	SAB.SW.1	RW.IM0.1
SAB.NE.2	RW.IM0.3	SAB.SW.2	RW.IM0.3
SAB.NE.3	RW.IM0.3	SAB.SW.3	RW.IM0.3
SAB.NE.4	RW.IM0.3	SAB.SW.4	RW.IM0.3
SAB.NE.5	RW.IM0.4	SAB.SW.5	RW.IM0.4
SAB.NE.6	RW.IM0.4	SAB.SW.6	RW.IM0.4
SAB.NE.7	RW.IM0.4	SAB.SW.7	RW.IM0.4
SAB.NE.8	RW.IM0.4	SAB.SW.8	RW.IM0.4
SAB.NE.9	RW.IM0.4	SAB.SW.9	RW.IM0.4
SAB.NE.10	RW.IM0.4	SAB.SW.10	RW.IM0.4
SAB.NE.11	RW.IM0.4		
SAB.NE.12	RW.IM0.4		
SAB.NE.13	RW.IM0.4		

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5. WASTEWATER NETWORK

5.1. Design criteria

The domestic wastewater network was designed considering the following assumptions:

- use of HDPE corrugated pipes SN 8 kN/m^2;
- all manhole in the project will have a strength class of D400;
- main pipelines will have a minimum longitudinal slope of not less than 1%;
- pipe diameter: DN 200 minimum for final section at sewer inlet;
- velocities between 0.5 and 2 m/s;
- all pipes will be fixed to the manholes with EMACO-type mortar to guarantee the tightness of the entire manhole-pipe line;
- the preliminary steady flow sizing of the network assumed a maximum fill rate equal to 75%.

The positioning of the sewage manholes outside the buildings have been assumed by the buildings' drainage design. The design flow rates are not yet available, except for the first-flush (SPP) network. Consequently, at this stage no hydraulic sizing of the network is performed: this document presents only the design criteria, the principal network elements, and the plan layout (3567_6992_T01_Rev0_Layout-Networks and hydraulic works). All quantitative checks will be developed from the external manholes downstream once the final data are received.

The wastewater collection system is divided into three distinct network types: the civil wastewater network, the industrial wastewater network, and the first-flush wastewater network. The first network includes a siphonic manhole at the head of the system. The three discharge types are kept separate and then follow the scheme presented in Chapter 3 (Figure 3.2: Planimetric location of discharge point S2 in the WebGIS Acque di Lombardia) for discharge.

The positioning of the sewage manholes on the outside of the buildings and the design flow rates have been assumed by the drainage design of the buildings, and all the calculations set out below are valid starting from the manholes on the outside of the buildings and continuing downstream.

The wastewater collection system is divided into three distinct network types: the civil wastewater network, the industrial wastewater network, and the first-flush wastewater network. The three discharge types are kept separate; each stream passes through its final sampling manhole before mixing, in accordance with the scheme in Chapter 3 (Figure 3.3), and ultimately discharges at point S2 on Via Liguria.

The civil wastewater network consists of a single pipeline, responsible for collecting wastewater at the exit of the main building. The pipeline (SAN) will be sized when the civil discharge from the building will be available.

The industrial wastewater network consists of a single pipeline, that collects condensate water from the main building and the draining waters from the tank serving the fire-fighting system. The pipeline (SAI) will be sized when the civil discharge from the substation will be available.

First-flush water coming from the generator/fuel storage are and, from the loading/unloading bays are conveyed to the treatment systems and then discharged through a pumping system.

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5.2. Inspection/connection manholes of sewage network

Connecting manhole in reinforced concrete complete with cast-iron manhole covers are provided.

The dimensions of the inner base of the inspection and connection manholes along the network are chosen depending on the pipe size, according to the following criteria:

- 0.4 x 0.4 m if the diameter of the inlet/outlet pipes does not exceed 200 mm;
- 0.5 x 0.5 m if the diameter of the inlet/outlet pipes exceeds 200 mm and does not exceed 300 mm;
- 0.6 x 0,6 m if the diameter of the inlet/outlet pipework is greater than 300 mm and does not exceed 350 mm;
- 0.8 x 0.8 m if the diameter of the inlet/outlet pipes is greater than 350 mm and does not exceed 500 mm;
- 1 x 1 m if the diameter of the inlet/outlet pipes is greater than 500 mm and not exceeding 700 mm;
- 1.2 x 1.2 m if the diameter of the inlet/outlet pipework is greater than 700 mm and not exceeding 900 mm;
- 1.5 x 1.5 m if the diameter of the inlet/outlet pipes is greater than 900 mm.

The selection of manhole types for the wastewater network is deferred to the next design phase and will be defined once the network has been hydraulically sized.

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